Flexural behaviour of steel fibre reinforced self-compacting concrete laminar structures

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ABSTRACT

A high strength Steel Fibre Reinforced Self-Compacting Concrete (SFRSCC) of high ductility was developed. To evaluate the contribution of steel fibre reinforcement for the flexural resistance of laminar structures, an experimental program was carried out with slab strips of distinct longitudinal reinforcement ratio (ρ_{sl}), submitted to four point loads. A total of twelve slab strips were tested, grouped in three series of distinct ρ_{sl} (0.2, 0.36 and 0.56). Each series is composed of four slab strips, two of them without fibre reinforcement, and the other two with 45 kg of hooked end steel fibres per cubic meter of concrete. From the force-deflection relationship the contribution of steel fibres for the slab load carrying capacity at the serviceability and ultimate limit states was assessed. An equivalence between the content of steel fibres and the percentage of a virtual conventional longitudinal steel bars was established. Taking the experimental results and performing an inverse analysis, a stress-strain diagram was obtained to characterize the post-cracking behaviour of the developed SFRSCC. The present work describes the experimental and the numerical research carried out, and presents and analyzes the main obtained results.

1. INTRODUCTION

The simulation of the contribution of steel fibres for the behaviour of Steel Fibre Reinforced Concrete (SFRC) laminar structures failing in bending is still a challenge since several parameters influence this contribution, such as: geometrical characteristics and material properties of the fibres; concrete properties; method of SFRC application; geometry of the structure, etc. Due to this fact, there are several approaches for modelling the fibres' contribution, which, in a certain way, does not contribute for a more extended use of this high performance material, even in certain applications where its use would result in technical and economic advantages.

The designers of reinforced concrete structures are still not well familiarized with SFRC and, when they need to design a SFRC structure, the most current question is the following: what percentage of longitudinal tensile steel bars can a certain content of fibres replace? This attitude results from the understanding that the simplest strategy for modelling the contribution of fibres for the flexural resistance of laminar concrete structures is to convert fibres in a certain percentage of a fictitious longitudinal conventional reinforcement. In the present work this strategy is explored for steel fibre reinforced self-compacting concrete (SFRSCC) laminar structures.

Self-Compacting Concrete (SCC) can be defined as a concrete that is able to flow in the interior of the formwork, filling it in a natural manner and passing through the reinforcing bars and other obstacles, flowing and consolidating under the action of its own weight (Okamura 1997). The advantages associated to the addition of steel fibres to concrete mixes may be joined with the ones resulting from the self-compactability concept in concrete, with the formulation of steel fibre reinforced concrete mixes exhibiting self-compacting ability. The resulting material is, in this work, designated by Steel Fibre Reinforced Self Compacting Concrete (SFRSCC) and, when compared to conventional concretes, presents clear technical advantages in terms of costs/benefits ratio (Barros *et al.* 2005). Previous research showed that the addition of steel fibres is especially favourable in structures of redundant supports, since the ultimate load carrying capacity of this type of structures is much higher than the crack corresponding to the concrete crack initiation (Barros and Figueiras 1998, Falkner and Teutsch 1993).

In the ambit of assessing the potentialities of SFRSCC for structures with redundant supports, an experimental program was carried out. In this type of structures, concrete crack control is a mandatory requisite in their design and, due to the congestion of conventional reinforcement in certain zones, its replacement for a certain content of steel fibres could be a favourable option in terms of assuring good concrete casting. The obtained results were used, not only to appraise the contribution of the steel fibres for the flexural behaviour of laminar structures, but also to propose a simple strategy to convert a certain content of fibres in an equivalent fictitious longitudinal reinforcement. The SFRSCC was developed for Box-Culvert structures in order to assure the self-compacting requirements and the mechanical properties required by this underground structure. To assess the real benefits provided by fibre reinforcement mechanisms for this type of structures, the test set up should contemplate the possibility of simulating the stress redistribution that occurs in structures of high redundant supports, such is the case of Box-Culverts. However, this type of tests is too expensive and valid indicators can be obtained from much simpler tests like the four point bending test used in the present experimental program. Therefore, the experimental program of the present work is composed of 12 slab strips reinforced with distinct percentage of conventional longitudinal reinforcement and a constant content of hooked-ends steel fibres.

Previous research showed that fibre reinforcement efficacy is so high as more resistant is the micro-structure of the fibre-matrix interface, as long as fibres do not rupture (Gopalaratnam and Shah 1987). For Box-Culverts structures the required level of resistance was achieved developing a SCC of high compacity and reinforced with 45 kg of hooked ends steel fibres per m^3 of concrete of high aspect ratio (length/diameter) and high tensile strength (Barros *et al.* 2006). This SCC composition was used on the manufacture of the slab strips tested in the ambit of the present work.

2. EXPERIMENTAL PROGRAM

2.1 Series of tests

The experimental program is composed of three series of slab strips. Each slab has a shear span (a = 450 mm) almost equal to 3.5 times the effective depth of the slab cross section ($a/d \approx 3.5$), a total length of 1600 mm, a distance between supports of 1350 mm and a cross section of 350×150 mm^2 (Figure 1). A distinct percentage of longitudinal reinforcement was adopted for each series of slabs: 3ϕ6 (ρ_{sl} =0.2), 3ϕ8 (ρ_{sl} =0.36) and 3ϕ10 (ρ_{sl} =0.56), having been attributed the designation of A, B and C for these series, respectively. In all tested slabs, three steel bars of 6 mm diameter were used in the top part of the cross section. Twelve slabs were tested, six of them reinforced with steel fibres and the results of the others six were used for comparison purposes.

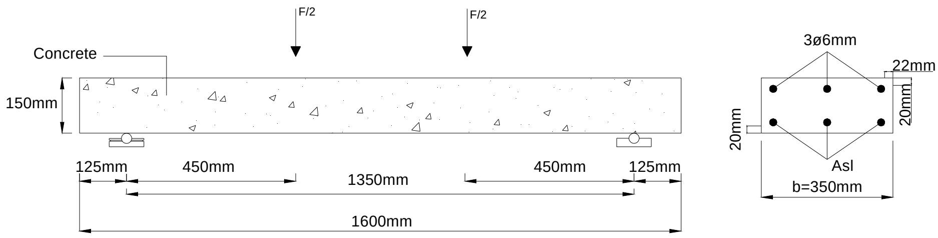

Figure 1 - Typical slab strip of the tested series.

2.2 Steel fibre reinforced self-compacting concrete mix composition

The mix composition adopted for the manufacture of the SFRSCC was optimized for a solid skeleton that includes 45 kg of the selected steel fibres per m^3 of concrete. This mix composition was obtained applying a design method that takes into account the strong perturbation effect produced by steel fibres on the flowability of fresh concrete. In fact, steel fibres are rigid and, consequently, do not easily accommodate to the dynamically changing shape of the bulk paste located between the particles constituting the granular skeleton structure. Consequently, the design procedure and the optimization process followed to achieve self-compactability requirements are sensible to the fibre content, as well as to the geometrical and material properties of the fibres (Pereira 2006, Barros *et al*. 2007).

The characteristics of the SFRSCC mix composition are included in Table 1. The materials used were cement (C) CEM I 52.5R (rapid hardening and high strength cement, according to EN197-1:2000), limestone filler MICRO 100 AB (LF), a superplasticizer (SP) with the tradename SIKA 3002 HE, water (W), four types of aggregates (fine river sand, FS; coarse river sand, CS; crushed calcareous 6-14 mm, CG_1; and 14-20, CG_2) and DRAMIX® RC-80/60-BN hooked end steel fibres (SF). This fibre has a length (l_f) of 60 mm, a diameter (d_f) of 0.75 mm, an aspect ratio (l_f / d_f) of 80 and a yield stress of 1100 MPa.

In the slab strips only reinforced with conventional steel bars, the mix composition of the used SCC was similar to the mix composition applied in the SFRSCC slabs, a part the fact of the former one do not include fibres. The authors are aware that this is not the most appropriate procedure, since the composition depends on the interferences introduces by the fibres. However, for the relatively small content of 45 kg per m^3 of concrete it was assumed that the changes necessary to introduce due to the presence of fibres are not so significant that compromise the principal conclusions of the present work. For both compositions the total spread, s, and the time to reach a spread diameter of 500 mm, T_f, measured in the Slump flow in conjunction with J Ring, were measured, as well as the H2/H1 (blocking ratio) parameter of the L Box test (EFNARC 2002). The obtained results are indicated in Table 1, showing that the self-compacting requirements were assured. For the developed compositions, no visual sign of segregation was detected and the mixtures showed good homogeneity and cohesion, even while flowing trough the smaller orifice of the Abrams cone (while testing, the Abrams cone was always used in the inverted position).

Table 1. Adopted composition (per m^3)

Designation	C (kg)	W (kg)	SP (dm^3)	LF (kg)	FS (kg)	CS (kg)	CG_1 (kg)	CG_2 (kg)	SF (kg)	s (mm)	T_f (s)	H2/H1
SCC	380.5	102.7	12.5	360.0	391.4	429.1	336.9	298.2	0	700	12	0.80
SFRSCC	380.5	102.7	12.5	360.0	391.4	429.1	336.9	298.2	45.0	710	16	0.77

2.3 Properties of the materials
2.3.1 Concrete
To assess the compressive strength of the SCC and SFRSCC, f_c, direct compression tests were carried out with cubic specimens of 150 mm edge. Using the recommendations of CEB-FIP (1993), the average and the characteristic values of the compressive strength at 28 days are those included in Table 2.

Table 2. Compressive strength of SCC and SFRSCC.

Designation	$f_{cm\ (t=28\ days)}$ (MPa)	f_{ck} (MPa)	Strength class
SCC	94.34	86.34	C70/85
SFRSCC	99.22	91.22	C70/85

2.3.2 Steel bars
The tensile behaviour of the steel bars used in the experimental program was characterized from uniaxial tensile tests. Based on the obtained stress-displacement curves, the values of the properties indicated in Table 3 were determined. The average value of the elasticity modulus of the steel specimens was about 200 GPa.

Table 3. Values of the main properties of the tested steel bars.

Diameter (mm)	Yield strain (ε_{sy}) (‰)	Yield stress (σ_{sy}) (MPa)	Maximum strain (ε_{su}) (‰)	Maximum stress (σ_{su}) (MPa)
6	2.8	568	10.0	605
8	3.0	585	14.3	635
10	3.0	591	11.5	625

3. TESTS AND RESULTS

3.1 Test setup
The slab strips were subjected to four line loads (distributed in the width of the slab cross section), see Figure 2. The force was registered by a load cell of 300 kN capacity, while the deflections were measured from five LVDT's (*Linear Voltage Differential Transducer*), two of them of a measuring length (l_{meas}) of 25 mm placed at the extremities of the slab, and the others three of l_{meas} = 50 mm placed at the central part of the slab (pure bending zone). To avoid the recordings of extraneous deflections, like support settlements, the LVDTs were supported on a Japanese Yoke bar, as represented in Figure 2. The tests were carried out with servo-controlled equipment, imposing a deflection ratio of 30 μm/s in the central LVDT for the test control purposes.

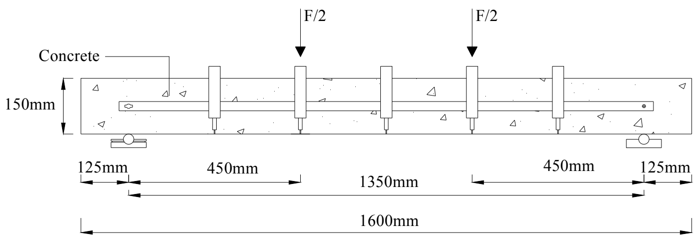

Figure 2 - Support, load conditions and monitoring system.

3.2 Results

A label Li_j_k was used to differentiate the tested slabs strips, where: "L" can be replaced by A, B or C to designate the series that the slab pertains; "i" identifies the number of the slab test in each series (two slabs were tested per each series); "j" represents the diameter of the steel bars used as tensile longitudinal reinforcement; "k" represents the quantity of applied fibres (value in kg per m^3). For instance, A2_6_45 slab represents the second slab of A series that is reinforced with 6 mm diameter longitudinal steel bars and includes 45 kg of steel fibres per concrete m^3. If "i" is omitted, the result represents the average value of the results of the slabs of the corresponding series. The principal characteristics of each slab are indicated in Table 4.

The relationship between the force and the midspan deflection of the tested slabs are represented in Figures 3, 4 and 5. Each curve corresponds to the average load values observed at each deflection level, obtained from the two slabs of each sub-series (with a constant Q_f). From the analysis of these curves it is observed that, after crack initiation, the load carrying capacity of the SFRSCC slabs is higher than the corresponding SCC slabs. The difference of the load carrying capacity between these two types of slabs increases from the crack initiation up to the maximum load of the SFRSCC slab. However, this difference decreases with the increase of the percentage of the conventional reinforcement.

Table 4. Principal characteristics of the tested slabs.

Designation	Longitudinal reinforcement (A_{sl}^+)	Percentage of longitudinal reinforcement (ρ_{sl}*)	Content of fibres Q_f (kg/m^3)	Series
A1_6_0 A2_6_0	3Ø 6	0.20	0	A
A1_6_45 A2_6_45	3Ø 6	0.20	45	
B1_8_0 B2_8_0	3Ø 8	0.36	0	B
B1_8_45 B2_8_45	3Ø 8	0.36	45	
C1_10_0 C2_10_0	3Ø 10	0.56	0	C
C1_10_45 C2_10_45	3Ø 10	0.56	45	

$\rho_{sl}*=Asl^+/(b.d)*100$

In terms of serviceability limit states for deflection control, the Eurocode 2 (2004) recommends that the maximum deflection of a structural member should not exceed a limit value in the range [L/250-L/500], depending on type of structure, where L is the span length of the member, in mm. Assuming a limit value of $u = L/400 = 3.4$ mm for the deflection, the corresponding force (F_{ELUt}) was obtained (see Table 5). From the analysis of these values it can be verified that fibres increased F_{ELUt} from 1.29 up to 1.77, when the F_{ELUt} values of the SCC slabs are taken for comparison purposes. This increase was as higher as lower was the reinforcement ratio of the longitudinal steel bars, ρ_{sl}. This table also includes the values of the maximum forces supported by the tested slabs (F_{max}). From the analysis of the F_{ELUt} and F_{max} values it can be concluded that F_{ELUt}/F_{max} ratio ranged from 0.66 to 0.95 in the slabs without fibres, while in the slabs reinforced with fibres the F_{ELUt}/F_{max} ratio varied from 0.70 to 0.97. For both types of reinforcement F_{ELUt}/F_{max} ratio increased with the decrease of ρ_{sl}. This shows that fibre reinforcement is very effective for the verifications of the design requirements imposed by the serviceability limit states, being this effectiveness as more pronounced as lower is ρ_{sl}.

Table 5. Force values of the tested slabs.

Series	ρ_{sl}	F_{ELUt} (kN) F_{ELUt}^{SCC} [0 kg/m³]	F_{ELUt}^{SFRSCC} [45 kg/m³]	$\frac{F_{ELUt}^{SFRSCC}}{F_{ELUt}^{SCC}}$	F_{max} (kN) $F_{\max}^{SCC}$ [0 kg/m³]	$F_{\max}^{SFRSCC}$ [45 kg/m³]	$\frac{F_{\max}^{SFRSCC}}{F_{\max}^{SCC}}$	$\frac{F_{ELUt}^{SCC}}{F_{\max}^{SCC}}$	$\frac{F_{ELUt}^{SFRSCC}}{F_{\max}^{SFRSCC}}$
A	0.20	27.68	49.03	1.77	29.19	50.76	1.74	0.95	0.97
B	0.36	42.46	64.45	1.52	55.42	78.96	1.42	0.77	0.82
C	0.56	55.53	71.60	1.29	83.53	102.59	1.23	0.66	0.70

To evaluate the increase in terms of slab's load carrying capacity provided by fibre reinforcement during the deflection process of the slabs, the difference between the load carrying capacity of the SFRSCC and SCC slabs, ΔF, for each deflection value, was evaluated. The relationship between ΔF/F ratio and the slab midspan deflection is represented in Figure 6, in which F is the load carrying capacity of the SCC slab at the same deflection where ΔF is evaluated. From the resulting curves it is apparent that the contribution of the fibres for the slab load carrying capacity starts from very early stages of the slab deformation, just after the formation of incipient cracks. It is visible that ΔF/F increases up to the deflection corresponding to F_{max} of the SFRSCC slabs, having this increase attained a maximum value of 80%. The decrease of ΔF/F ratio with the increase of ρ_{sl} is also apparent. For the slabs reinforced with the minimum percentage of longitudinal reinforcement (L_6) the maximum value of the ΔF/F ratio occurred at a deflection of about the deflection corresponding to the serviceability limit state (u_{ELUt} = 3.4 mm). In the series of slabs reinforced with the other two percentages of longitudinal reinforcement, the ΔF/F ratio maintained almost constant in the deflection range between 20% and 200% of the u_{ELUt}. This means that the benefits provided by fibre reinforcement for the serviceability limit states are as more pronounced as lower is the longitudinal reinforcement ratio. In case of necessity to increase ΔF/F of slabs of considerable ρ_{sl}, a higher content of fibres needs to be applied. However, economic and technical aspects should be considered since, besides the higher costs of the fibres (in comparison to the one of conventional steel bars), it should be also taken into account the costs derived from the necessity of using higher percentage of fine materials in the concrete composition when the content of fibres increases.

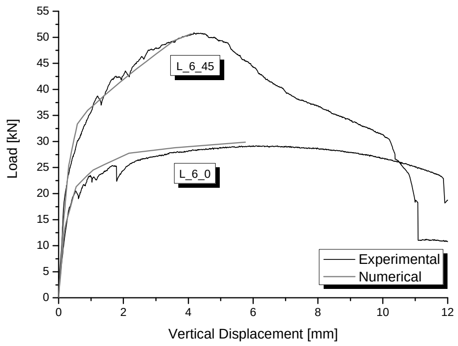

Figure 3 - Load-central deflection curves of serie L_6.

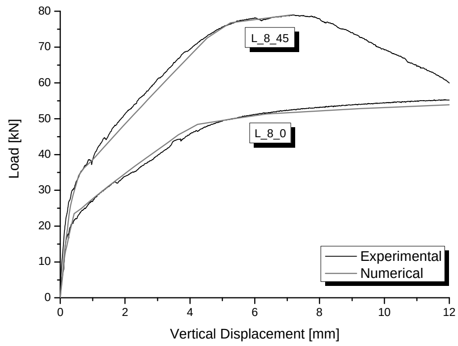

Figure 4 - Load-central deflection curves of serie L_8.

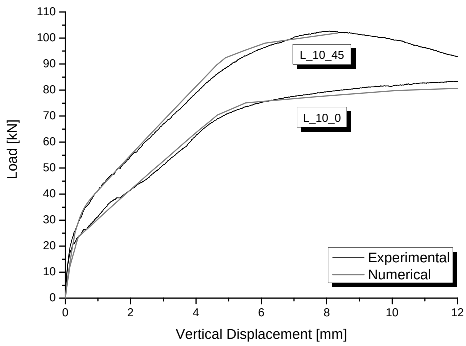

Figure 5 - Load-central deflection curves of serie L_10.

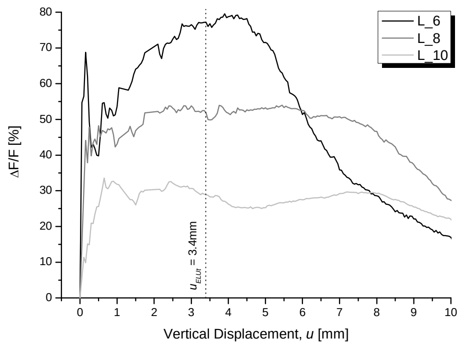

Figure 6 - Relationship between the midspan deflection and the relative increment of the slab load carrying capacity for all series.

The indices $I_{F(ELUt)}$ and $I_{F(Max)}$, representing the relative increase of the slab load carrying capacity provided by fibre reinforcement for the deflection corresponding to the serviceability limit states and for the deflection corresponding to the maximum load carrying capacity of SFRSCC slab, respectively, were determined from (1) and (2).

$$I_{F(ELUt)} = \frac{F_{ELUt}^{SFRSCC} - F_{ELUt}^{SCC}}{F_{ELUt}^{SCC}} \times 100 \; ; \; I_{F(Max)} = \frac{F_{Max}^{SFRSCC} - F_{Max}^{SCC}}{F_{Max}^{SCC}} \times 100 \qquad (1,2)$$

where F_{ELUt}^{SFRSCC}, F_{ELUt}^{SCC} and F_{Max}^{SFRSCC}, F_{Max}^{SCC} are the forces of the SFRSCC and SCC slabs at u_{ELUt} and at peak load, respectively. The values of $I_{F(ELUt)}$ and $I_{F(Max)}$ are included in Table 6.

Table 6. Increase provided by fibre reinforcement in terms of load carrying capacity at serviceability ($I_{F(ELUt)}$) and ultimate states ($I_{F(Max)}$).

Designation	$I_{F(ELUt)}$ (%)	$I_{F(Max)}$ (%)
L_6	77.13	73.90
L_8	51.79	42.48
L_10	28.94	22.82

Table 6 indicate that the reinforcement of 45 kg of fibres per m^3 of concrete provided a gain in the load carrying capacity at the deflection corresponding to the serviceability limit states that ranged from 29% to 77%. This gain decreased with the increase of ρ_{sl}. In terms of the maximum load of the slabs, this tendency was almost the same, since this gain varied from 23% to 74%, with an increase of the gain with the decrease of ρ_{sl}.

4. FIBRE DESTRIBUTION

To evaluate the degree of heterogeneity on the fibre distribution in the in-plane of the slab strip, three core samples were extracted along the longitudinal axis of the slab (Figure 8). The fibre distribution in the depth of the slab was also estimated cutting these samples in three slices of equal thickness.

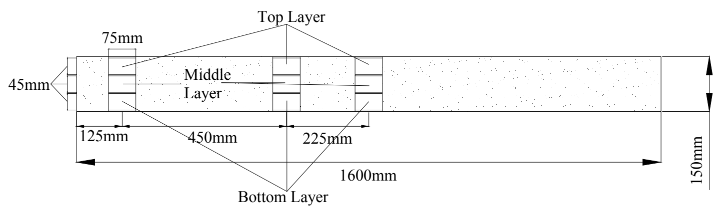

Figure 8 - Locations of the core samples extracted.

According to the EFNARC (1996), the amount of fibres is calculated from (3).

$$Q_f = \frac{m_f \times 1000}{V_c} \tag{3}$$

where m_f is the weight, in grams, of the extracted fibres from the core sample and V_c is the volume of the core sample in cm^3. As shown in Figure 9 the amount of fibres increasead in the depth of the slab, showing that, even without external compactation, the highest specific weight of the steel fibres, amongst the concrete constituents, led to a tendency of an increase of the fibre content with the depth of the laminar structural element. However, in the plan of the laminar structures fibres distribution does not show any tendency.

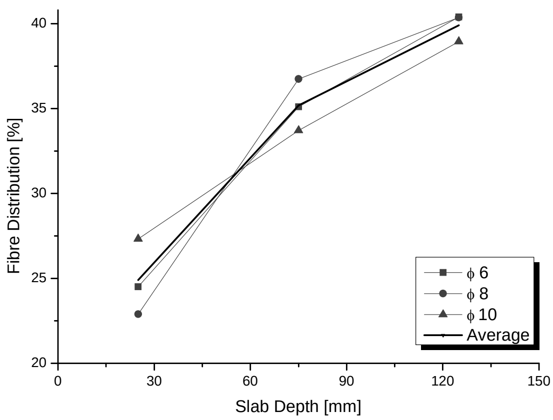

Figure 9 - Fibre distribution along the depth of the slab.

5. EQUIVALENCE BETWEEN CONTENT OF FIBRES AND PERCENTAGE OF A FICTITIOUS CONVENTIONAL REINFORCEMENT

To quantify the effect of the fibre reinforcement in terms of the increase of the flexural resistant moment of the slab, a methodology similar to the one used in the reinforced concrete design practice was adopted. According to this strategy, the design value of the resistant bending moment, M_{Rd}^{anal}, is calculated and compared with the value obtained from the experimental tests, $M_{R,\max}^{exp}$. Following the design practice of reinforced concrete linear members, M_{Rd}^{anal} can be determined from the following equation:

$$M_{Rd}^{anal} = \mu \times b \times d^2 \times f_{cd} \quad [\text{N/mm}^2] \tag{4}$$

where μ is the reduced bending moment of the cross section, which can be obtained from the mechanical percentage of the conventional reinforcement (Lima et al. 1985):

$$\omega = \frac{A_{sl}}{b\,d}\frac{f_{syd}}{f_{cd}} \tag{5}$$

where b is the width of the cross section of the element and d its effective depth, f_{cd} is the design value of the concrete compressive strength, A_{sl} , is the cross section area of the tensile longitudinal reinforcement and f_{syd} the design value of the yield stress of this reinforcement. Values of $f_{cd} = 51$MPa and $f_{syd} = 435$ MPa were obtained from the results registered in the experimental tests carried out with concrete and steel specimens (see Tables 2 and 3).

The value of M_{Rd}^{anal} was compared to the value of the bending moment recorded at the maximum load of the tested slabs, $M_{R,\max}^{exp}$, having been obtained the gain in terms of flexural resistance, ΔM_{flex}, that is calculated from the following equation:

$$\Delta M_{flex} = \frac{M_{R,\max}^{\exp} - M_{Rd}^{anal}}{M_{Rd}^{anal}} \times 100 \tag{6}$$

The values of ΔM_{flex} and the values of the corresponding $M_{R,\max}^{exp} / M_{Rd}^{anal}$ ratio are included in Table 7.

Table 7. Influence of the content of fibres on the flexural resistance.

ρ_{sl} (%)	M_{Rd}^{anal} [kN.m]	$M_{R,\max}^{exp}$ [kN.m]	ΔM_{flex} [%]	$M_{R,\max}^{exp} / M_{Rd}^{anal}$
0.20	4.68	11.42	144	2.44
0.36	8.35	17.77	113	2.13
0.56	12.48	23.08	85	1.85

The values indicated in Table 7 show that 45 kg of fibres per m^3 of concrete provided an increase of the resistance bending moment varying between 85% and 144%. It is also verified that ΔM_{flex} decreased with the increase of ρ_{sl} , but, even for the slabs with the highest ρ_{sl} the increase still was significant. Following this methodology, the 45 kg of fibres per m^3 of concrete used in the present experimental program can be converted in a given percentage of a fictitious conventional reinforcement, placed at the level of the existent tensile longitudinal bars. Table 8 includes the obtained results, where $A_{f,eq(s)}^{\max}$ is the area of the cross section of this fictitious reinforcement. In this evaluation a value of 1.3 was adopted for the partial safety factor for FRC, γ_f, which is recommended by the Italian CNR-DT 204/2006 document, when high quality control on materials as well on structures is assured, since the developed SFRSCC it will be used in pre-casting industry.

Table 8. Equivalence between a cross section area of a fictitious conventional reinforcement and a content of steel fibres.

ρ_{sl} [%]	M_{Rd}^{anal} [kN.m]	$M_{R,\max}^{exp}/\gamma_f$ [kN.m]	$A_{f,eq(s)}^{\max}$ [mm^2]	$\frac{A_{f,eq(s)}^{\max}}{A_{sl}}\times 100$ [%]
0.20	4.68	8.78	159	187
0.36	8.35	13.67	238	158
0.56	12.48	17.75	311	132

The values of Table 8 indicate that 45 kg per m^3 concrete of the steel fibres used in the present experimental program are equivalent to a fictitious reinforcement, which, in terms of cross section area and comparing to the cross section area of the steel bars really applied, varies between 186% for the series with ρ_{sl} = 0.20 up to 132% in the series with ρ_{sl} = 0.56.

6. NUMERICAL SIMULATION

Previous works (Barros et al. 2006) have shown that, using a cross-section layered model that takes into account the constitutive laws of the intervenient materials, and the cinematic and the equilibrium conditions, the deformational behaviour of structural elements failing in bending can be predicted from the moment-curvature relation, M-χ, of the representative sections. To verify the capability of this model for predicting the deformational behaviour of SFRSCC laminar structures, the tests carried out in the ambit of the present work were simulated. To evaluate the M-χ of the cross sections, a discretization in 50 layers of 3 mm thickness was used. The tensile and compression longitudinal reinforcements were converted in steel layers with a thickness that provides the cross section area of the corresponding steel bars, placed at 20 mm and at 130 mm from bottom surface of the cross section (see Figure 1). The concrete properties of the constitutive model are included in Table 2, while the steel properties are indicated in Table 3. The concrete strain-softening behaviour is simulated by the trilinear stress-strain diagram. From this inverse analysis the concrete fracture parameters can be assessed (see Table 9).

6. CONCLUSIONS

The present work resumes the main results obtained in an experimental program carried out to assess the effect of a constant content of steel fibre reinforcement (45 kg/m^3) for the behaviour of reinforced self-compacting concrete laminar structures failing in bending. For this purpose three series of slab strips reinforced with distinct percentage of conventional longitudinal steel bars were tested under four line loads.

Table 9. Concrete properties.

Panel	Compression		Tension	Softening				
	f_{ck} (MPa)	E_{ci}^{a} (GPa)	σ_{cr} (MPa)	ε_1 (‰)	σ_1 (MPa)	ε_2 (‰)	σ_2 (MPa)	G_f (N/mm)
L_6_0	86.34	40.00	3.00	2.00	0.20	4.00	0.10	0.080
L_8_0	86.34	40.00	3.00	2.30	0.45	6.00	0.42	0.092
L_10_0	86.34	40.00	3.00	2.30	0.45	6.00	0.41	0.085
L_6_45	91.22	45.00	3.50	5.00	0.65	40.00	0.45	3.900
L_8_45	91.22	45.00	3.50	5.00	0.65	80.00	0.57	3.900
L_10_45	91.22	45.00	3.50	5.00	0.65	60.00	0.43	3.900

Using a content of cement of 380 kg per cubic meter of concrete, a SFRSCC of strength class C70/85 was produced, which shows that the mix design method developed in previous research is able of developing high strength SFRSCC at a competitive cost.

The force-deflection relationships obtained in the slabs tests evinced that 45 kg per m^3 of concrete of the selected fibres contributed significantly for the slab load carrying capacity from the incipient crack formation stage, i.e., from a deflection level that is lower than 1/10 of the deflection corresponding to the serviceability limit states. The increase in terms of slab load carrying capacity provided by fibre reinforcement was as significant as lower was the percentage of the conventional reinforcement (ρ_{sl}). At the deflection corresponding to the serviceability limit states, 45 kg of the used steel fibres per m^3 of concrete provided an increase of the slab load carrying capacity that ranged from 29% for ρ_{sl} = 0.56 up to 77% for ρ_{sl} = 0.2.

In terms of the slab maximum load carrying capacity, this content of steel fibres provided an increase that varied from 23% for ρ_{sl} = 0.56 up to 74% for ρ_{sl} = 0.36.

Applying a methodology used in the design practice of reinforced concrete structures an equivalence between the content of fibres and a cross section area of a fictitious longitudinal reinforcement (A_{sl}^{fict}) was determined. From the obtained results it was verified that 45 kg per m^3 of concrete is equivalent, in terms of flexural resistance, to a A_{sl}^{fict} of 74 mm^2, 87 mm^2 e 75 mm^2 for the slabs with ρ_{sl} =0.20, 0.36 e 0.56, respectively, which corresponds to a 87%, 58% e 32% of the cross sectional area of the real reinforcement applied in the tested slabs.

A cross section layer model was used to determine the moment-curvature relationship, M-χ, of the representative sections of the tested slab strips. The M-χ was used to evaluate the tangential flexural stiffness, $(EI)_T$, during the panel loading process. The tangential stiffness matrix of the panel was evaluated from the $(EI)_T$ of each element discretizing the panel, and using the framework of the matrix displacement method. This simple numerical strategy was able to predict, with enough accuracy, the load-deflection response registered experimentally.

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